TOOPS

Investigation of a Steel Railway Viaduct

Civil Engineering

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INVESTIGATION

OF A

STEEL RAILWAY VIADUCT

BY

GEORGE NOBLE TOOPS

THESIS

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U N I V E R S I T Y   O F   I L L I N O I S

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This is to certify that the thesis prepared under the immediate direction of Assistant Professor F. O. Dufour by

GEORGE NOBLE TOOPS

entitled INVESTIGATION OF A STEEL VIADUCT

is approved by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Ira O. Baker.

Head of Department of Civil Engineering

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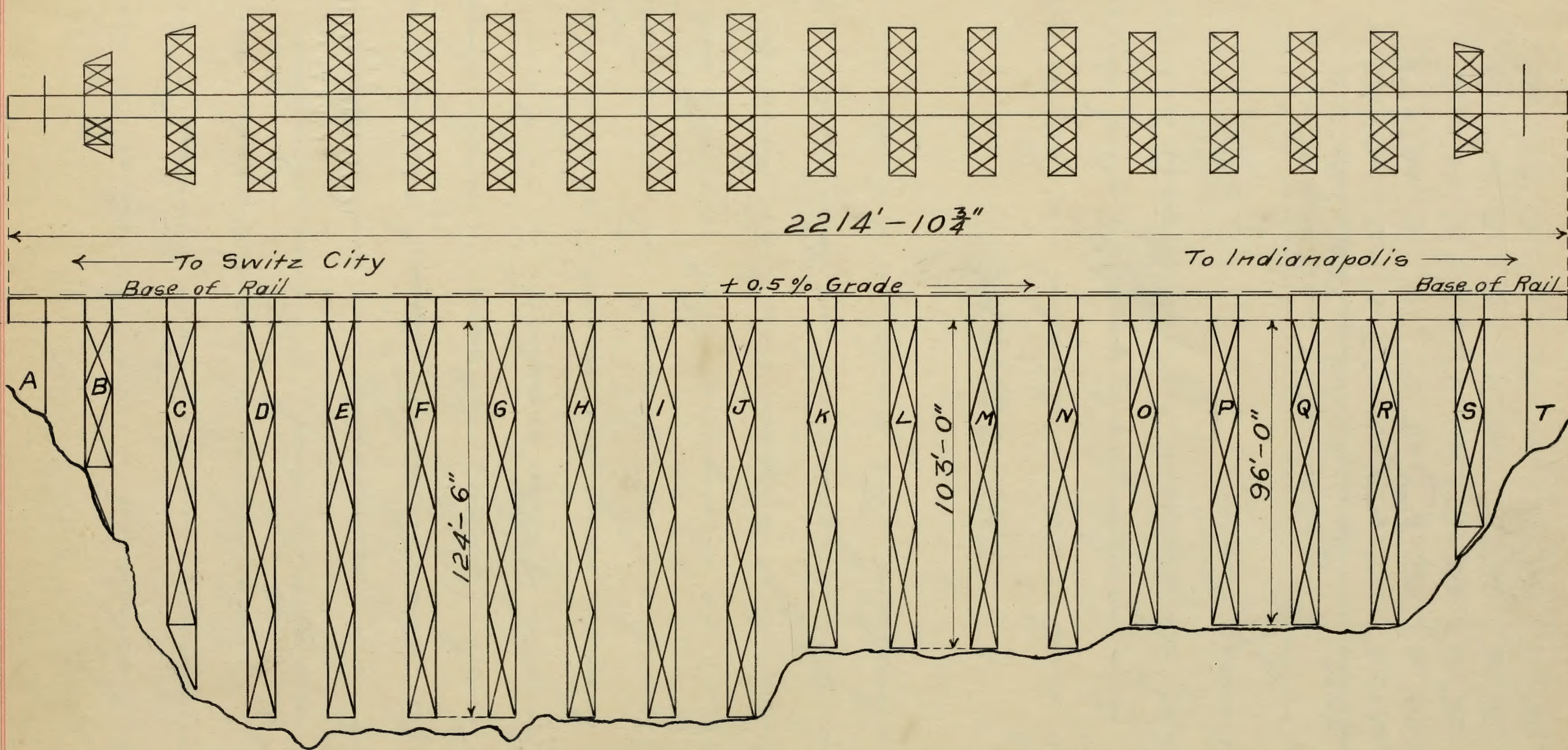


STEEL VIADUCT

OVER

RICHLAND CREEK, GREEN CO., IND.

ON THE

INDIANAPOLIS SOUTHERN RY.

LOCATION

The Richland Creek Viaduct is located on the Indianapolis Southern Railroad, 21 miles northeast of Switz City, Ind. and 71 miles southwest of Indianapolis, Ind. The construction of this viaduct was necessary in order that the Indianapolis Southern Railroad could cross Richland Creek Valley, Green County, Indiana. The total length of the viaduct is 2214 feet 10 $\frac{3}{4}$ inches; the towers vary in height from 46 feet to 124 feet 6 inches. At present it is single-track but it is so designed that by the addition of two outside legs to each bent it can be made double-track. The alignment is tangent over the entire structure; the grade is 0.5 per cent.

THE TOWERS.

The girders are supported by two concrete abutments, two rocker bents, and 18 steel towers. The principal dimensions of the towers are shown in the following table:

No. of Towers	Height	Longitudinal		Transverse	
		Top	Bottom	Top	Bottom
7	124'-6"	40'-0"	40'-0"	8'-0"	54'-8$\frac{1}{4}$"
4	103'-0"	40'-0"	40'-0"	8'-0"	46'-7$\frac{1}{2}$"
5	96'-0"	40'-0"	40'-0"	8'-0"	44'-0"
1	76'-6"	40'-0"	40'-0"	8'-0"	36'-8$\frac{1}{4}$"
1	46'-0"	40'-0"	40'-0"	8'-0"	33'-10$\frac{1}{2}$"

III. The transverse and longitudinal bracing of the towers consists of two 10" x 20 lb. channels with double lacing bars $2\frac{1}{2}$" x $\frac{3}{8}$". There are no horizontal struts except at the top and bottom of the transverse, and at the bottom of the longitudinal bracing, the girders acting as a strut at the top of the latter. All the struts have the same section as the bracing. In the tallest towers the longitudinal bracing consists of four panels, and the transverse of six. The heights of these are shown on the ~~stress~~ sheet. From tops of the columns down to a splice near the center, the section consists of four angles 4"x4"x $\frac{9}{16}$" and two plates 21"x $\frac{1}{2}$", and from there to their base the section is made up of four angles 4"x4"x $\frac{5}{8}$" and two plates 21"x $\frac{5}{8}$" with double lacing bars of 3"x2"x $\frac{3}{8}$" angles.

THE GIRDERS.

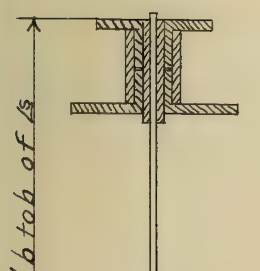


Fig. 1

The girders are spaced 8 feet center-to-centers of webs and have a depth of 7'-6" back-to back of angles. The top and bottom flanges of the girders are made up of side plates and angles. In the construction of the top flange, two side plates $10\frac{3}{4}$" x $\frac{3}{8}$" extending almost the full length of the girder and

four angles 6"x4"x$\frac{3}{4}$", are used. The top set of angles of this flange has the 6-inch leg turned to the web, while the bottom set has the 4-inch leg turned to the web. In the bottom flange, two plates 9$\frac{3}{4}$"x$\frac{3}{8}$" and two angles 6"x6"x$\frac{3}{4}$" are used. The long girders weigh 40,000 lb. (for drawing of girder see page 23.).

INVESTIGATION

In the design of this structure the American Bridge Company's specifications were used, with one exception; the live load and wind stresses being determined according to Cooper's 1901 specifications, the live load used being his standard E 50 loading.

This investigation will consist of an investigation of one of the 75-foot girders, the transverse and longitudinal bracing and columns of one of the tallest towers, and of the rocker bent supporting the ends of the two 60-foot girders

WIND STRESSES

These stresses were determined graphically. The complete solution is given on Plate I, p 5.

WIND STRESSES IN DIAGONALS TOWER 'D'

	TRAIN ON	TRAIN OFF		TRAIN ON	TRAIN OFF
EG	23500	-25000	EG	+23500	+25000
GH	+21000	+23300	GH	-20000	-21200
HI	-15300	-17000	H'I'	+15300	+18800
IJ	+13000	+16000	I'J'	-11000	-13200
JK	- 9000	-11200	J'K'	+10700	+15000
KY	+10500	+16000	K'Y'	- 7600	-10800

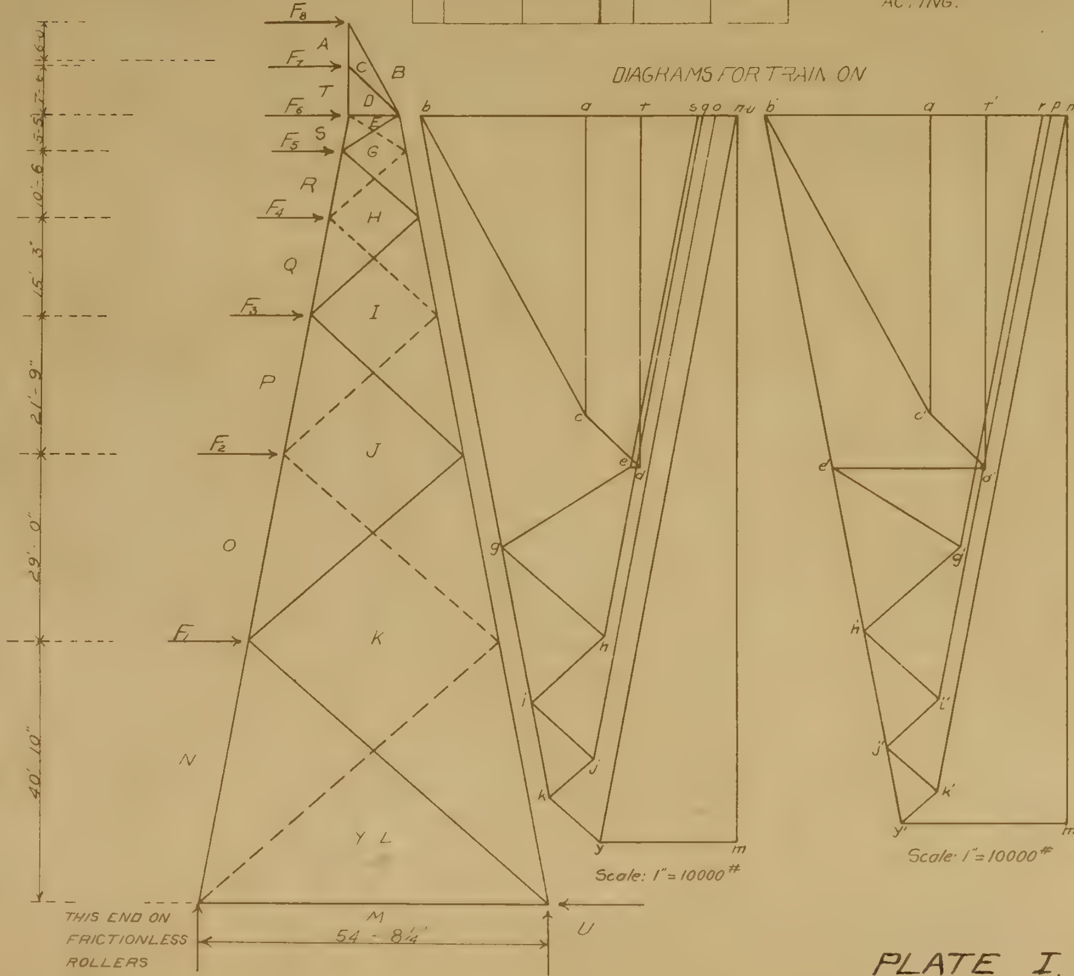
NOTE:

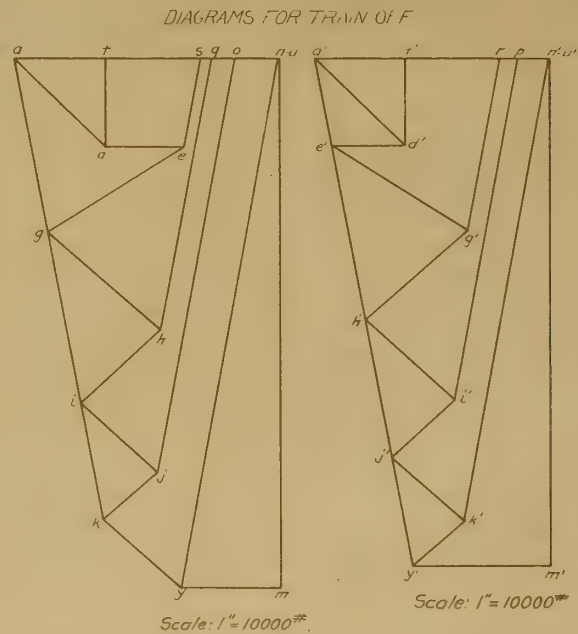
WITH TRAIN ON IT WAS ASSUMED THAT EACH SYSTEM OF BRACING TOOK STRESSES DUE TO FORCES F_6, F_7, F_8 AND THEIR RESPECTIVE FORCES DOWN SIDE OF TOWER; SAME FOR TRAIN OFF, F_8 NOT ACTING.

WIND FORCES COOPER'S SPECIFICATIONS

	TRAIN ON			TRAIN OFF	
F_8	$\frac{40+15}{2}\times 450$	25900	F_8		
F_7	$\frac{40+15}{2}\times 150$	8600	F_7	$\frac{15+40}{4}\times 500$	14380
F_6	$\frac{40+15}{2}\times 150+\frac{5.5}{2}\times 100$	8900	F_6	$\frac{15+40}{4}\times 500+\frac{5.5}{2}\times 200$	[illegible]900
F_5	$\frac{5.5+10.5}{2}\times 100$	800	F_5	$\frac{5.5+10.5}{2}\times 200$	1600
F_4	$\frac{10.5+15.25}{2}\times 100$	1300	F_4	$\frac{10.5+15.25}{2}\times 200$	2500
F_3	$\frac{15.25+21.75}{2}\times 100$	1850	F_3	$\frac{15.25+21.75}{2}\times 200$	3700
F_2	$\frac{21.75+29}{2}\times 100$	2540	F_2	$\frac{21.75+29}{2}\times 200$	5100
F_1	$\frac{29+40.83}{2}\times 100$	3490	F_1	$\frac{29+40.83}{2}\times 200$	7000

Scale 1" = 10'





WIND STRESS DIAGRAMS FOR TOWER "D"
OF
RICHLAND CREEK VIADUCT
ON THE
INDIANAPOLIS SOUTHERN RAILWAY

PLATE I.

LIVE LOAD STRESSES.

On account of the probability that at some future time this viaduct will be changed into a double-track structure, Fig. 2 page 8, it was necessary to design the transverse bracing of the towers for live load stresses also. The live load which must be considered is that which will come on cap A when the structure is double-tracked; and will be the maximum end shear for a 75-foot girder under Cooper's E 50 loading. This equals 147,100 lb.

As there are three members and one known force meeting at cap A, the ordinary methods of statics for the determination of stresses can not be applied, consequently, the method of Least Work must be used.

A complete analysis of the method of Least Work can be found in Chapter XV, Johnson's Modern Framed Structures. In determining the live load stresses in the transverse bracing by the method of Least Work, the following formulea were used.

$$S_r = -\frac{\sum_1^n \frac{S'ul}{aE}}{\frac{l_r}{a_r E_r} + \sum_1^n \frac{u^2 l}{aE}}.$$

$$S = S' + S_r u.$$

Where:

S' = stress in any member due to live load when redundant member is left out.

u = factor of reduction = stress in any member due to 1 lb compression in redundant member.

E = modulus of elasticity of any member.

E_r = " " " " redundant member.

l = length of any member in inches.

l_r = " " redundant member " "

a = area of any member.

a_r = " " redundant member.

S_r = stress in redundant member.

S = true stress in any member.

AC was assumed as the redundant member. The solution of the live load stresses is given on Plate II, p. 9.

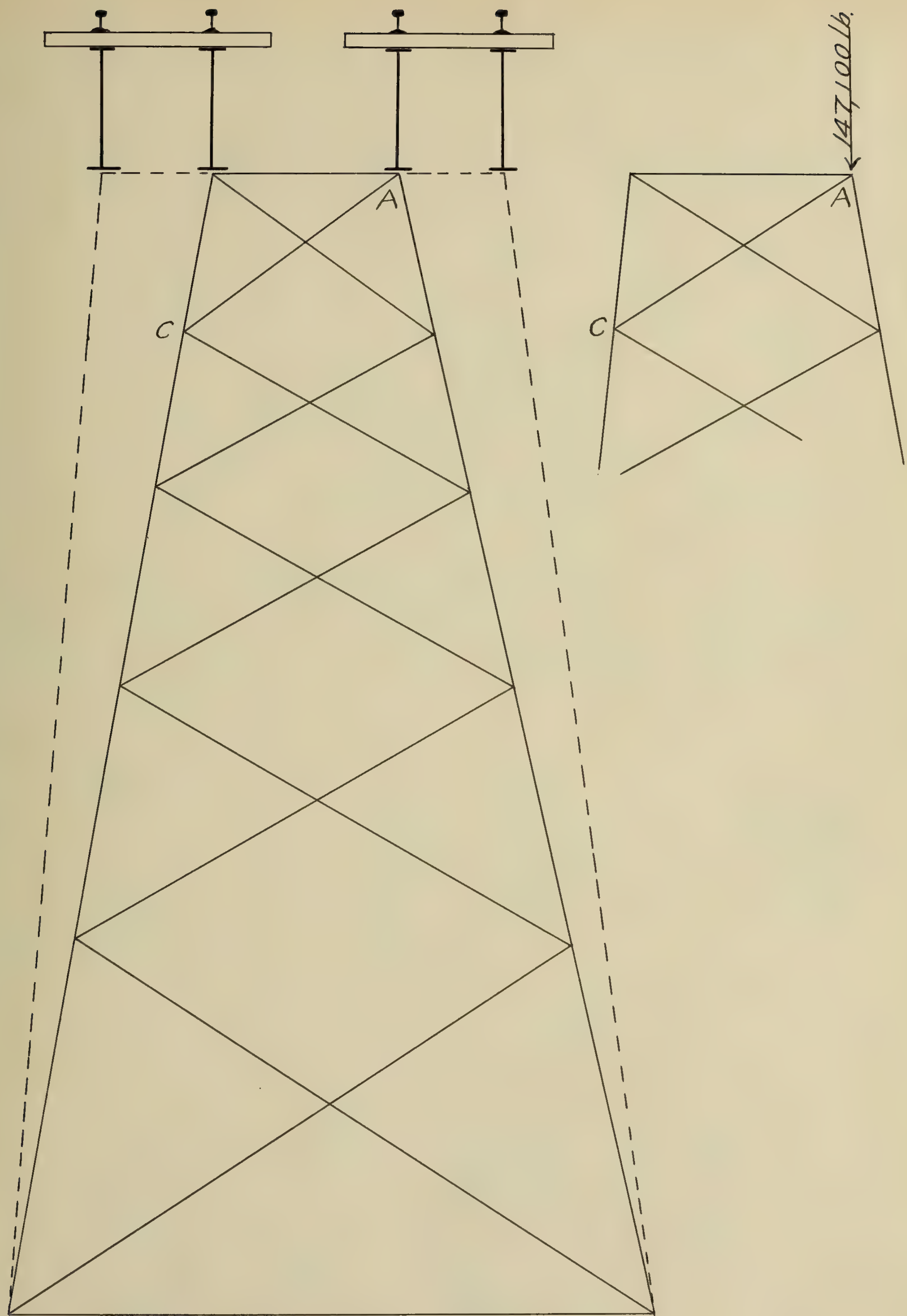
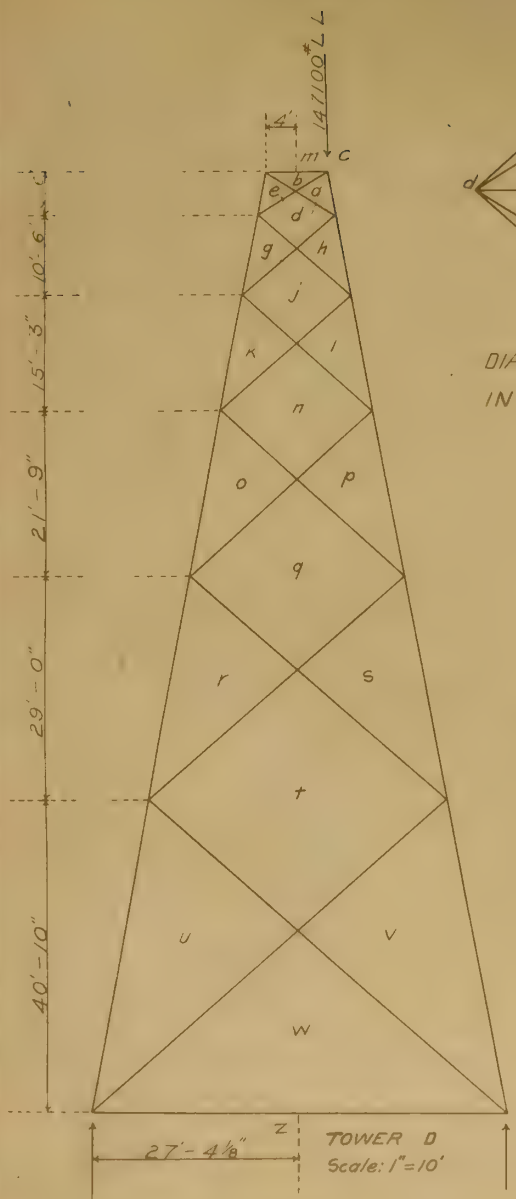


Fig. 2. TRANSVERSE BRACING.



TOWER D
Scale: 1"=10'

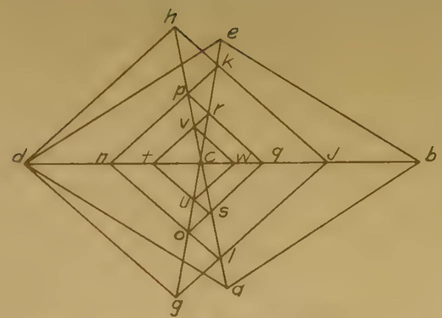


DIAGRAM FOR 1# COMPRESSION
IN REDUNDANT MEMBER "edorba"
Scale: 3"=1#

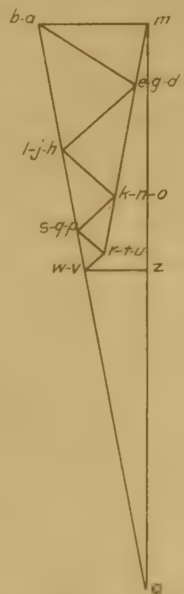


DIAGRAM FOR LIVE LOAD
Scale: 1"=20000#

PLATE II

STRESSES BY LEAST WORK

Member	l	a	S'	u	$\frac{ul}{a}$	$\frac{S'ul}{a}$	$\frac{u^2l}{a}$	$S_r u$	S
ca	68	37.72	+150000	-.55	-1.00	-150000	+.549	-8530	+141470
bc-m	96	11.76	+27000	-.95	-7.75	-209000	+7.35	-14700	+12300
be	128	11.76	-29000	+1.00	+10.90	-360000	+10.90	+15500	-13500
ec-m	68	37.72	+16000	-.55	-1.00	-16000	+.549	-8530	+7470
ed									
cg-m	127	37.72	+16000	+.55	+1.85	+29600	+1.02	+8530	+24530
gd	192	11.76	000	-.90	-14.70	000	+13.20	-13950	-13950
dh	192	11.76	+25000	-.90	-14.70	-368000	+13.20	-13950	+11050
hc	127	37.72	+116000	+.60	+2.02	+236000	+1.22	+9300	+125800
ck-m	184	37.72	+46000	-.42	-2.25	-103500	+.94	-6500	+39500
kj	270	11.76	-18000	+.63	+14.45	-260000	+9.10	+9770	-8230
jl	270	11.76	000	+.63	+14.45	000	+9.10	+9770	+9770
lc	184	37.72	+116000	-.42	-2.25	-262000	+.94	-6500	+110000
co-m	264	37.72	+46000	+.30	+2.10	+96500	+.63	+4650	+50650
on	384	11.76	000	-.44	-14.35	000	+6.30	-6830	-6830
np	384	11.76	+13000	-.44	-14.35	-187000	+6.30	-6830	+6170
pc	264	37.72	+95000	+.30	+2.10	+199000	+.63	+4650	+99650
cr-m	353	44.64	+61000	-.20	-1.58	-96500	+.31	-3100	+57900
rq	528	11.76	-9000	+.30	+13.50	-121000	+4.05	+4650	-4350
qs	528	11.76	000	+.30	+13.50	000	+4.05	+4650	+4650
sc	353	44.64	+95000	-.20	-1.58	-150000	+.31	-3100	+91900
cu-m	418	44.64	+61000	+.17	+1.59	+97000	+.27	+2640	+63640
ut	744	11.76	000	-.23	-14.50	000	+3.34	-3570	-3570
tv	744	11.76	+7000	-.23	-14.50	-101500	+3.34	-3570	+3430
vc	418	44.64	+85000	+.17	+1.59	+135000	+.27	+2640	+87640
wz	656	11.76	-16000	+.13	+7.25	-116000	+.93	+2000	-14000
						-1698000	+98.80		

$\frac{l_r}{a_r} = \frac{128}{11.76} = 10.9$

$S_r = -\frac{-1698000}{98.80+10.9} = +15500$

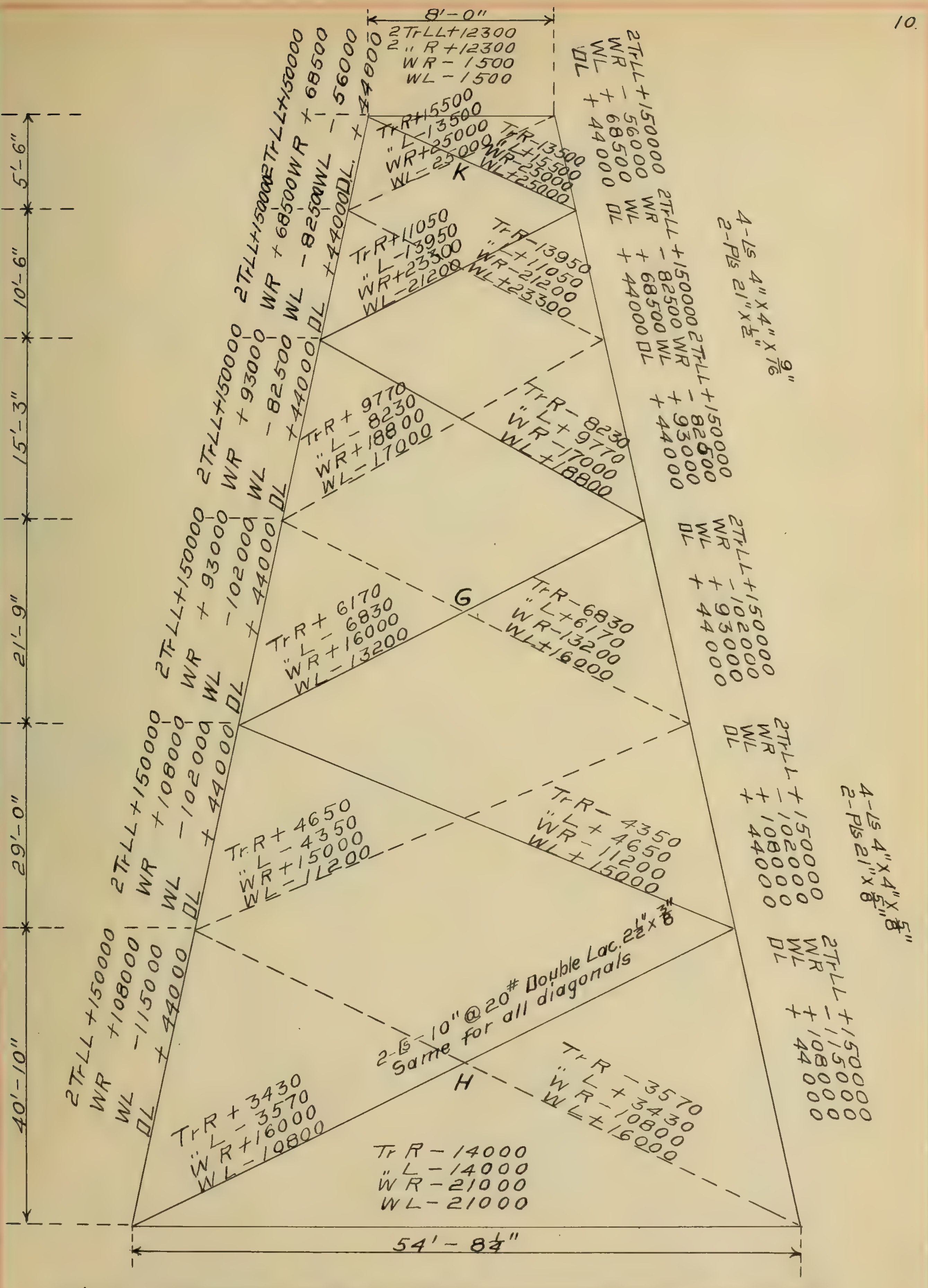


Fig. 3. STRESS SHEET FOR TRANSVERSE BRACING.

STRESSES DUE TO THE BRAKING OF THE TRAIN

According to the American Bridge Company's specifications, the longitudinal bracing of the trestle towers and similar structures must be designed to resist the momentum produced by suddenly braking of the train, the coefficient of friction of wheels sliding upon the rails being assumed as 0.2.

As both ends of the 40-foot and one end of the 75-foot girders are fixed, the maximum wheel load will be the sum of the separate wheel loads that are in the 40-foot and 75-foot girders when wheel two is placed at the end of the 40-foot girder, and is equal to 378,000 lb. The load acting on the tower due to the momentum is 378,000 X 0.2 = 75,000 lb.

Each system of longitudinal bracing is designed to take one half of the above load or 37,800 lb. The shear for each system of bracing will be 37,800 lb., and the stress in each brace equals the shear x Sec θ.

$$\operatorname{Sec}\theta = \frac{\sqrt{(28.5)^2+(40)^2}}{40} = 1.23$$

$$\operatorname{Sec}\theta_1 = \frac{\sqrt{(30.75)^2+(40)^2}}{40} = 1.26$$

$$\operatorname{Sec}\theta_2 = \frac{\sqrt{(32.83)^2+(40)^2}}{40} = 1.29$$

For stresses in bracing, see Fig. 5, p. 14.

The stresses in the columns will be determined by considering the bent as a lattice truss, and by dividing it into two separate systems, one containing columns and dotted diagonals, and the other, the columns and full diagonals. Each system is a Warren truss, and the stresses in the columns of each system can be found in the same way as chord stresses are found in the Warren truss. In order to determine the true stresses in any section of the columns, the sum of the stresses found for that section in the separate systems must be taken.

The stresses for each system are:

Full System

$U_0U_2 = -\dfrac{(37{,}800)(28.5)}{40} = -27{,}000$ $\qquad L_0L_2 = +\dfrac{(37{,}800)(28.5)}{40} = +27{,}000$

$L_1L_3 = +\dfrac{(37{,}800)(59.25)}{40} = +56{,}000$ $\qquad U_1U_3 = -\dfrac{(37{,}800)(59.25)}{40} = -56{,}000$

$U_2U_4 = -\dfrac{(37{,}800)\cdot(90)}{40} = -85{,}000$ $\qquad L_2L_4 = +\dfrac{(37{,}800)(90)}{40} = +85{,}000$

$L_3L_4 = +\dfrac{(37{,}800)(122.83)}{40} = +116{,}000$ $\qquad U_3U_4 = -\dfrac{(37{,}800)(122.83)}{40} = -116{,}000$

For drawings of separate systems see Fig. 4. p. 13, and for true column stresses see Fig 5 p. 14.

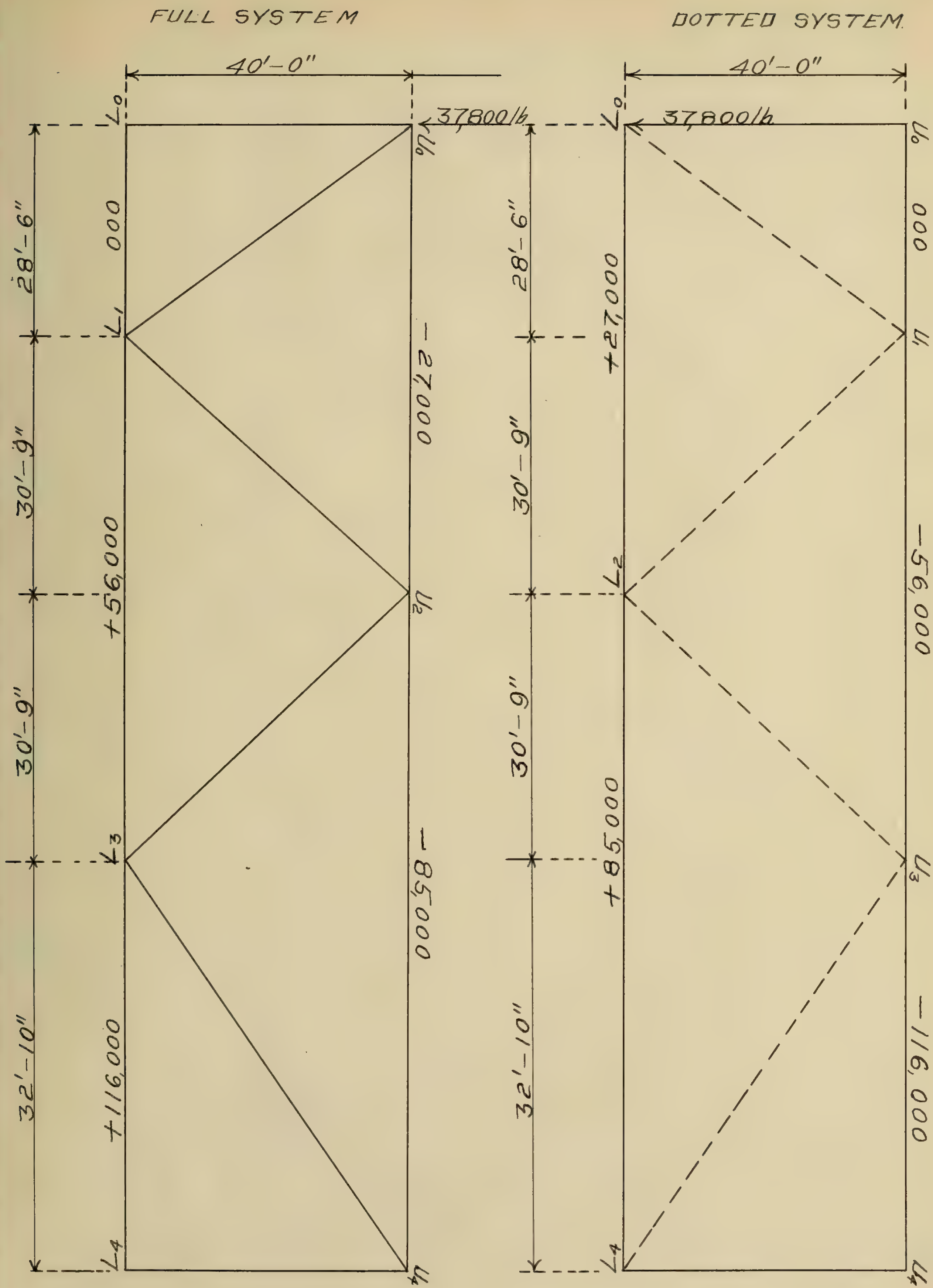


Fig. 4. BRAKING STRESSES, SEPARATE SYSTEMS

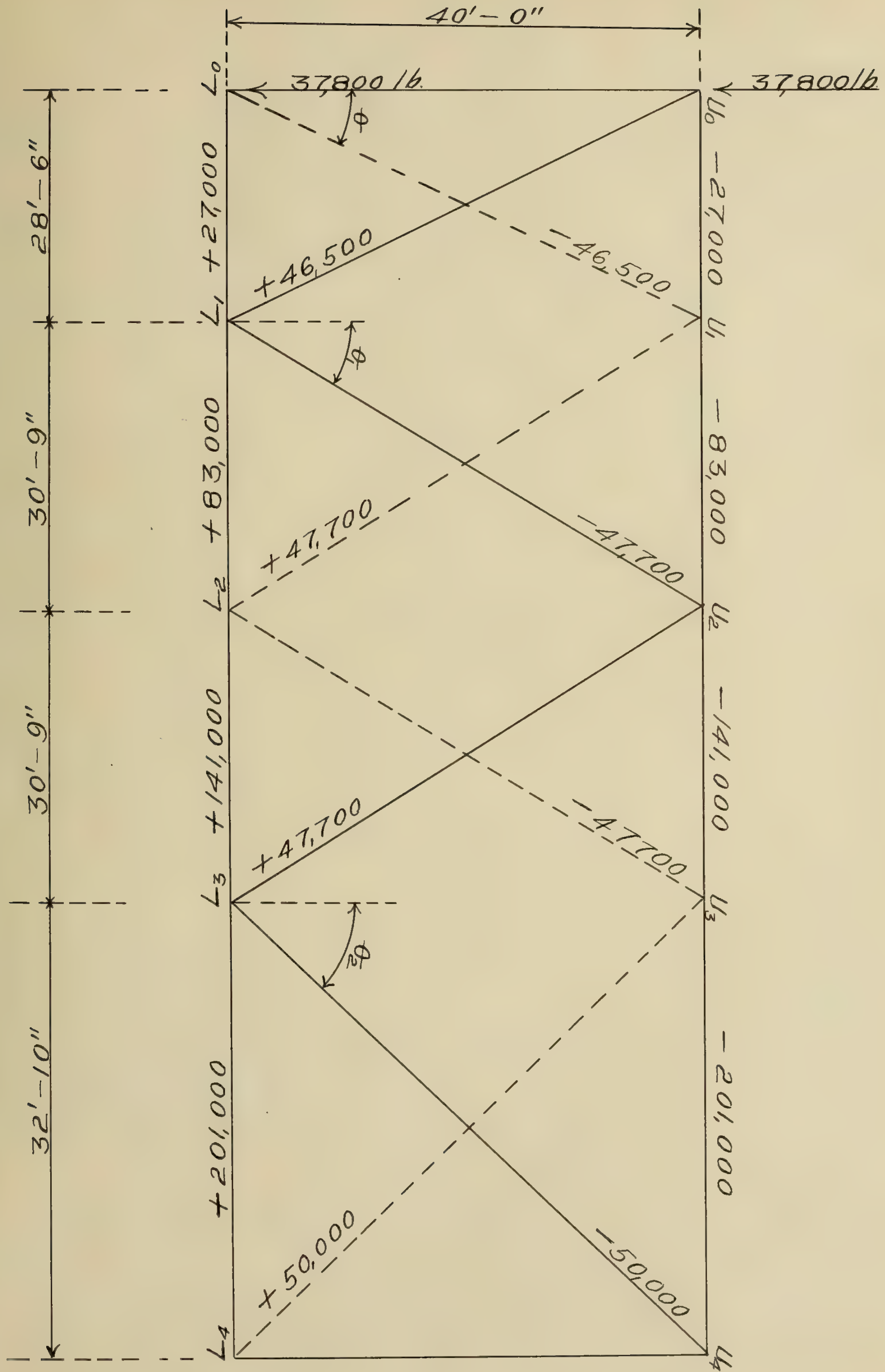


Fig 5. BRAKING STRESS SHEET

THE CROSS-SECTION OF THE MEMBERS.

Having determined all the stresses in the tower, an investigation will now be made of the cross-section of several of the members in order to see if they meet the requirements of the Specifications.

In this investigation of the cross-section of the transverse diagonals, it will be assumed that if two trains running in opposite directions at a fair rate of speed, pass upon the viaduct, alternate strains will be produced. Consequently according to Specifications the total sectional area of the members of the transverse bracing must be equal to the sum of the areas required for each strain. The investigation follows:

DIAGONAL K

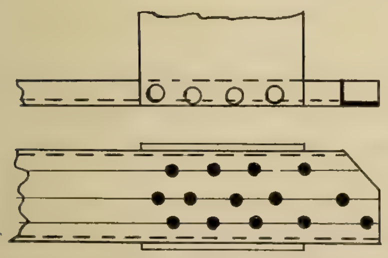


Fig. 6. RIVET SPACING IN END OF DIAGONALS

This member is located in the top panel of the transverse bracing, Fig. 3, and consists of two channels 10" x 20 lb. The member has a gross area of 11.76 sq. in.; length 128 inches, and the least radius of gyration of one of the channels is 3.66.[4] The maximum direct stresses in the member are 52,100 lb. compression and 38,500 lb. tension. These include stress due to impact. The unit stress, p, for tension = 15,000 lb. per sq. in., and

for compression, $r = 3.66$,[4] $l/r = 35$, and therefore, $p =$ 13,750 lb. per sq. in. The area required for tension is 38,500/15,000 = 2.58 sq. in. and for compression 52,100/13,750 = 3.77 sq. in. In the end connections of the member, Fig. 6, the rivets are spaced so three rivet-holes come in the same cross-section, consequently, 2.6 sq. in. must be deducted, making a total required area of 8.95 sq. in. The given area is 11.76 sq. in. which gives 31.4 % excess area in the member. The rivets are all $\frac{7}{8}$ inches in diameter, having a shearing stress of 6,610 lb., and the number required = 52,100/6,610 = 8 while 28 are used.

DIAGONAL G.

This member is located in the second panel of the transverse bracing, Fig. 3, having the same section as the one just investigated, and a length of 384 inches. The maximum direct stresses in the member are 26,800 lb compression and 25,100 lb. tension, each including the stresses due to impact. The unit stress, p, for tension = 15,000 lb per sq. in., and for compression, $r = 3.66$,[4] $l/r = 105$, and $p = 8,250$ lb. per sq. in. The area required for tension is 25,100/15,000 = 1.67 sq. in., and for compression 26,800/8,250 = 3.25 sq. in. The rivets in the end connections of this member are spaced the same as is shown in Fig 6, consequently 2.6 sq. in. must be deducted, making a total required area

of 7.52 sq. in. The given area is 11.76 sq. in. which gives 56.5 % excess area for the member. The number of rivets required is 5, while 28 are used.

DIAGONAL H.

This member is located in the bottom panel of the transverse bracing, Fig 3, having the same section as one just investigated, and a length of 744 inches. The maximum direct stresses in the member are 22,000 lb. compression and 17,000 lb. tension, each stress including stresses due to impact. The unit stress, p, for tension = 15,000 lb. per sq. in. and for compression, r = 3.66, l/r = 203 and p = 3,700 lb. per sq. in. The area required for tension is 17,000 / 15,000 = 1.13 sq. in. and for compression 22,000/3,700 = 5.95 sq. in. The rivets in the end connections of this member are spaced the same as is shown in Fig 6, consequently 2.6 sq. in. must be deducted, making a total required area of 9.68 sq. in. The given area is 11.76 sq. in. which gives 21.5 % excess area for the member.

THE CROSS-SECTION OF THE COLUMNS.

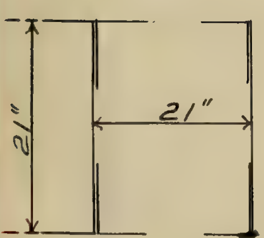


Fig 7.
Cross-section of Tower Columns

A section is taken through the column above the splice near the center of the tower. From the splice to the to the top of the tower the column consists of four angles 4"x 4" x $\frac{9}{16}$" and two

plates 21" × $\frac{9}{16}$". The length of the member is 264 inches. It has a gross area of 37.72 sq. in., and a maximum direct stress of 483,000 lb. compression, which includes stresses due to impact and the braking of the train. The least radius of gyration is $r = \sqrt{\frac{2110}{37.72}} = 7.70$" which for $l/r = 34.3$ gives a unit allowable stress for compression of 13,800 lb. per sq. in. The area required for compression = 48,3000/13,800 = 35.0 sq. in. while the given area is 37.72 sq. in., thus the member has 7.8% excess area.

From the splice near the center of the tower to the bottom, the column consists of four angles 4" × 4" × $\frac{5}{8}$" and two plates 21" × $\frac{5}{8}$". The length of the member is 498 inches. It has a gross area of 44.70 sq. in., and a maximum direct stress of 615,000 lb compression, which includes stresses due to impact and braking of the train. The least radius of gyration is $r = \sqrt{\frac{2571}{44.70}} = 7.58$" which for $l/r = 65.8$ gives a unit allowable stress for compression of 11,360 lb per sq. in. The area required for compression = 615,000/11,360 = 54.20 sq. in. thus showing a deficiency of 9.5 sq. in. in the cross-section of the member.

According to the last investigation there is not sufficient metal in the column be-

low the splice. However, in the above investigation 201,000 lb. compression due to the suddenly braking of the train was considered. In the second paragraph on page 433, Johnson's Modern Framed Structures, the following statement is made: "The longitudinal bracing of the trestle towers was proportioned to resist the strains resulting from a longitudinal force of 800 lb. per linear foot, but the columns were not increased in area on account of this force". The above statement referred to a series of viaducts which had been designed by the American Bridge Company.

THE CROSS-SECTION OF THE LONGITUDINAL BRACING.

It will only be necessary to investigate these members for the maximum compression stresses occurring in them. These stresses are shown on the stress sheet, Fig. 5 p. 14. All the members of the longitudinal bracing consists of two channels 10" x 20 lb. The number of rivets and their spacing is the same as that shown in Fig 6, p. 15. Reference to members will be made according to the notation employed on the stress-sheet, Fig 5. p. 14.

THE DIAGONAL $U_3 L_4$

This member is located in the bottom panel of the longitudinal bracing. It's length is 620 inches, gross area of 11.76 sq. in., and it has a maximum compressive strain of 50,000 lb. The unit stress for compression, $r = 3.66$, $l/r = 169$ is $p = 4,820$ lb. per sq. in. The area required for compression = 50,000/4,820 = 10.40 sq. in. The given area is 11.76 sq. in. which gives 13% excess area for the member. The number of rivets required = 50,000/6,610 = 8 while, 28 are used.

THE DIAGONAL $U_1 L_2$

This member of the longitudinal bracing is located in the second panel from the top. It's length is 605 inches, gross area of 11.76 sq. in. and it has a maximum compressive stress of 47,700 lb. The unit stress for compression, $r = 3.66$, $l/r = 165$ is $p = 4,980$ lb. per sq. in. The area required for compression = 47,700/4,980 = 9.60 sq. in. The given area is 11.76 sq in., which gives 22.5% excess area for the member. The number of rivets required = 47,700/6,610 = 7, while 28 are used.

THE DIAGONAL $U_0 L_1$

This member is located in the top panel of the longitudinal bracing. It's length is

590 inches, gross area 11.76 sq. in. and it has a maximum compression stress of 46,500 lb. The unit stress for compression, $r = 3.66$, and l/r being equal to 161 is 5,100 lb. per sq. in. The area required for compression = 46,500/5,100 = 9.13 sq. in. The given area is 11.76 sq. in. which gives 28.8 % excess area for the member. The number of rivets required = 46,500/6,610 = 7, while 28 are used.

INVESTIGATION OF THE 75-FOOT GIRDER.

An investigation of the flanges at the quarter and center points of the web and of the number and spacing of the rivets will be made. For drawing of the girder, and the shear and moment diagrams see Plate III, p. 23.

THE FLANGE AREA AT THE CENTER OF THE GIRDER

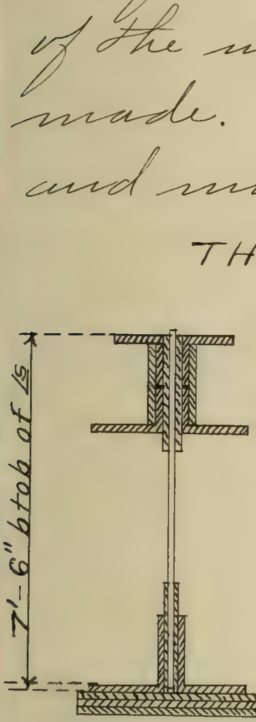


Fig. 8, CROSS SECTION OF GIRDER AT CENTER.

At this point the tension flange will be investigated. It consists of two angles 6"×6"×$\frac{3}{4}$", two plates $9\frac{3}{4}$"×$\frac{3}{8}$" and three cover plates each $14\frac{1}{2}$"×$\frac{1}{2}$", making a total net area of 39.17 sq. in. The distance from the back of the angles to the neutral axis of the section is $\bar{x} = \frac{13.88 \times 1.78 + 18.75 \times 6.54 \times 4.87}{39.17} =$ 1.8 inches, which gives an effective depth of $90 - 3.6 = 86.4$ inches. The maximum bending moment at the center of the girder is 56,700,000 in.-lb., and the net flange area required $= \frac{56,700,000}{15,000 \times 86.4} =$ 43.70 sq. in. According to the Specifications, one-eight of the web area or $\frac{90.25 \times 7}{8 \times 16} = 4.95$ sq. in. is to be considered as flange area. This leaves $(43.70 - 4.95)$ 38.75 sq. in. of flange area required in the angles and plates. The given net area of the angles and plates in the flange is 39.17 sq. in. This gives 1.8% of excess for the flange area.

L_0 — L_1 — L_2 — L_3 — L_4

18'-5¼" × 18'-5¼" × 18'-5¼" × 18'-5¼"

73'-9" C to C of Bearings

Used Engine Diagram Scale
Loading = Cooper's E 50

MOMENTS AND SHEARS

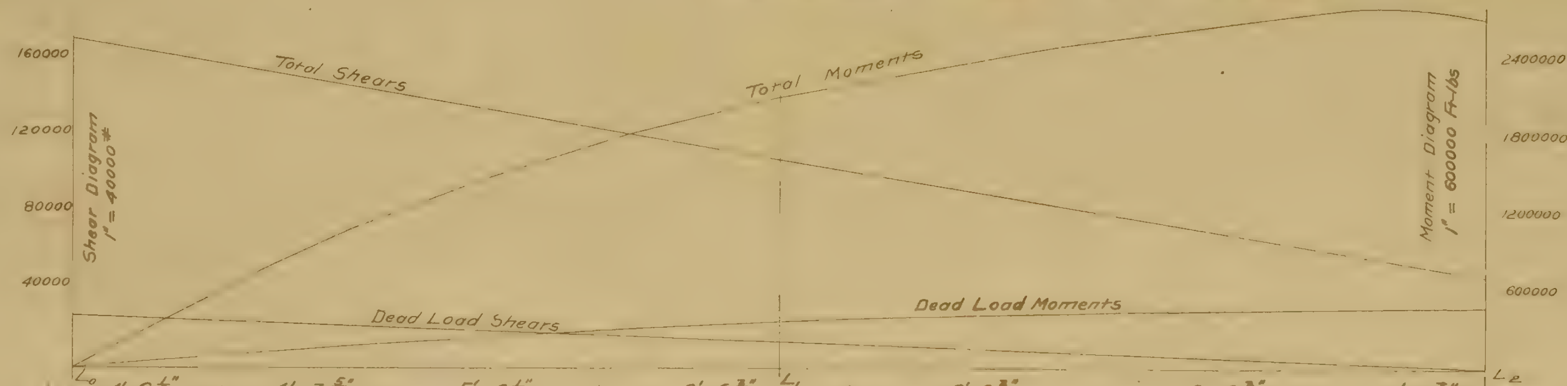
Point	DL-V's	+LL-V's	DL-MOM	LL-MOM
L_0	+26700	+145000	000	000
L_1	+13350	+95500	370000	1750500
L_2	000	+48600	492000	2240000
L_3	-13350	+14900	372000	1435000
			Max LL Mom	2335000

WHEELS FOR MAX SHEAR

Point	Wheel	G	$\frac{G}{n}$	$G+P_1$	Loaded Length
1	2	10	$\frac{152}{4}=38$	30	56
	(3)	30	$\frac{172}{4}=43$	50	64
2	2	10	$\frac{129}{4}=32.25$	30	48
	(3)	30	$\frac{142}{4}=35.5$	50	56
3	(2)	10	$\frac{90}{4}=22.5$	30	23
	3	30	$\frac{90}{4}=22.5$	50	23

WHEELS FOR MAX MOMENT

Point	Wheel	$\frac{G_1}{a}$	$\frac{G}{a}$	$\frac{G_1+P_1}{a}$	Loaded Length
1	2	10	$\frac{152}{4}=38$	30	56
	(3)	30	$\frac{172}{4}=43$	50	64
2	4	25	$\frac{142}{4}=35.5$	35	48
	(5)	35	$\frac{152}{4}=38$	45	56
	6	45	$\frac{172}{4}=43$	515	64
3	8	387	$\frac{152}{4}=38$	43	56
	(9)	43	$\frac{172}{4}=43$	474	64
	10	474	$\frac{212}{4}=53$	507	74



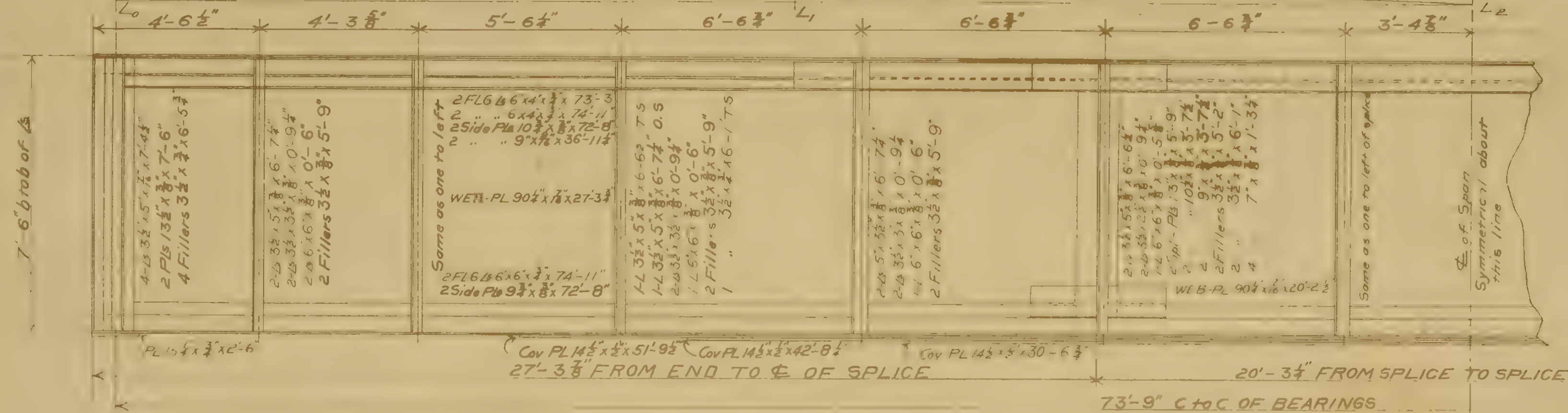


PLATE III

THE FLANGE AREA AT THE QUARTER-POINT

At this point the tension flange will be investigated. It has the same section as is shown in Fig. 8, p. 23, except there are only two cover plates in the bottom. The net area of the angles and plates is 32.92 sq. in. The distance from the back of the angles to the neutral axis of the section is $\bar{x} = \frac{13.88 \times 1.78 + 12.50 \times 0.5 + 6.54 \times 4.87}{32.92} = 1.91$ inches, which gives an effective depth of $90 - 3.82 = 86.18$ inches. The maximum moment at the quarter-point is 42,250,000 in.-lb., and the net flange area required $= \frac{42,250,000}{15,000 \times 86.18} = 32.70$ sq. in. As in the case at the center of the girder, one-eighth of the web area or 4.95 sq. in. is to be considered as flange area, which leaves $32.70 - 4.95 = 27.75$ sq. in. of flange area required in the angles and plates. The given net area in the angles and plates of the flange is 32.92 sq. in., which gives 15.7% of excess for the flange area.

WEB-PLATE.

The web-plate is $90\frac{1}{4}'' \times \frac{7}{16}''$ and has a gross area of 39.40 sq. in. According to the Specifications, the unit shearing stress for web-plates is 9,000 lb per sq. in. The maximum shear will be at the end of the girder, and is equal to 287,700 lb, which

includes the dead load shear and impact. The area required in the rub-plate = $\frac{287,700}{9,000}$ = 32.00 sq. in. The given area is 39.40 sq. in., which gives 19% excess of area.

THE RIVETS IN THE END STIFFENERS.

The number of rivets required in one pair of end stiffeners is equal to the maximum end shear divided by twice the bearing value of a $\frac{7}{8}$-inch rivet on a $\frac{7}{16}$-inch plate, or = $\frac{287,700}{2 \times 8430}$ = 16.7 = 17 rivets, while 18 are used.

THE RIVET SPACING IN THE FLANGES OF THE GIRDER

The rivet spacing in the flanges at the end, quarter, and center points of the girder will be investigated. The over-all height of the girder is 90 inches and the inside distance (h_g) = 73$\frac{3}{8}$ inches. An 8"×10" tie placed on edge will be used on the track, and they will be spaced with openings not exceeding 6 inches between ties. According to Cooper's 1901 Specifications, the maximum wheel load, 30,000 lb., on one driver, is to be distributed over three ties. This gives 715 lb. per linear inch of track. Thus it will be seen that the force acting on the rivets

in the flanges will be the resultant force (W) due to the wheel load and the stress in the flange angles. Therefore the rivet spacing $= \dfrac{v}{\sqrt{W^2 + \left(\frac{F}{F+\frac{A}{8}} \cdot \frac{V}{hg}\right)^2}}$ where v = the bearing value of a $\frac{7}{8}$ inch rivet on a $\frac{7}{16}$-inch plate, F = the flange area, A = web-area and V = total shear at the section. For the spacing of rivets at the the points in question see table below:

RIVET SPACING IN FLANGES.

Sec.	W	W^2	V^2	$\frac{F}{F+\frac{A}{8}}$	$\frac{V}{h_g}$	$\left(\frac{V}{h_g}\right)^2$	$\sqrt{W^2+\left(\frac{VF}{[F+\frac{A}{8}]h}\right)^2}$	v	Required Spacing	Given Spacing
End	715	511,200	287,700	.81	3,830	14,688,900	3,140	8,430	2.68	2
$\frac{1}{4}$	"	"	180,500	.87	2,460	11,971,600	3,050	"	2.76	3
$\frac{1}{2}$	"	"	85,200	.89	1,160	1,345,600	1,250	"	6.75	4

ROCKER BENT T.

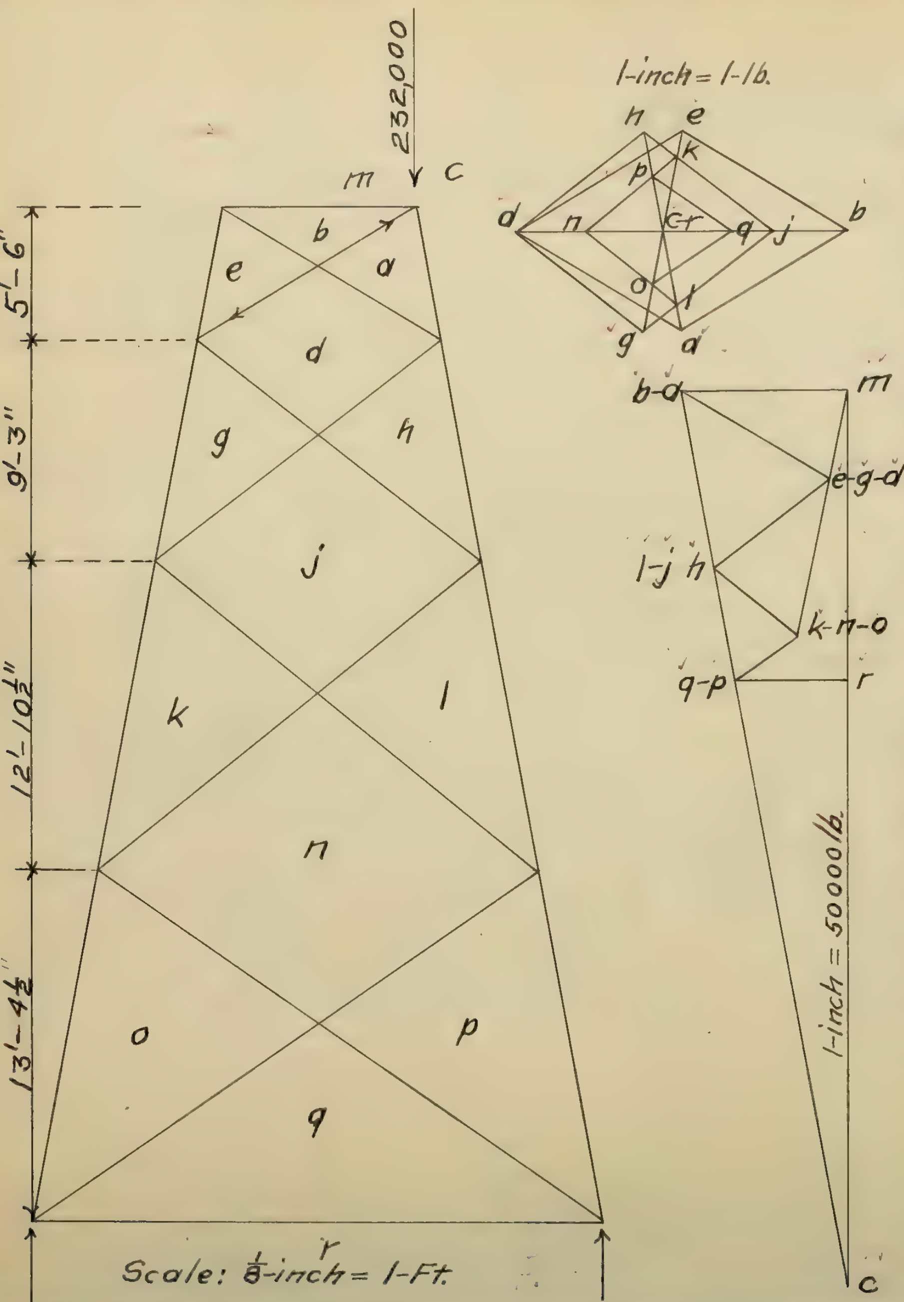
This bent is located at the northeast end of the viaduct, and supports the ends of two 60-foot girder spans. The investigation of it will be similar to that of the transverse bracing of the towers, the live load stresses in the bracing of the bent being determined by the method of Least Work. For the complete solution of the live load stresses see p. 28; also see p. 29 for the determination of the maximum wind stresses, the same assumptions being made here as in the investigation of the other bents and their bracing.

THE CROSS SECTION OF THE MEMBERS.

The columns will not be investiged as their unsupported lengths and maximum stresses are considerably less than those of the same cross-section in the large towers which were found to meet the requirements of the Specifications.

THE DIAGONAL K

This member is located in the top panel of the bracing of the bent, Fig. 10, and consists of two channels 10" x 20 lb. The member has a gross area of 11.76 sq.in.; length of 128 inches, and

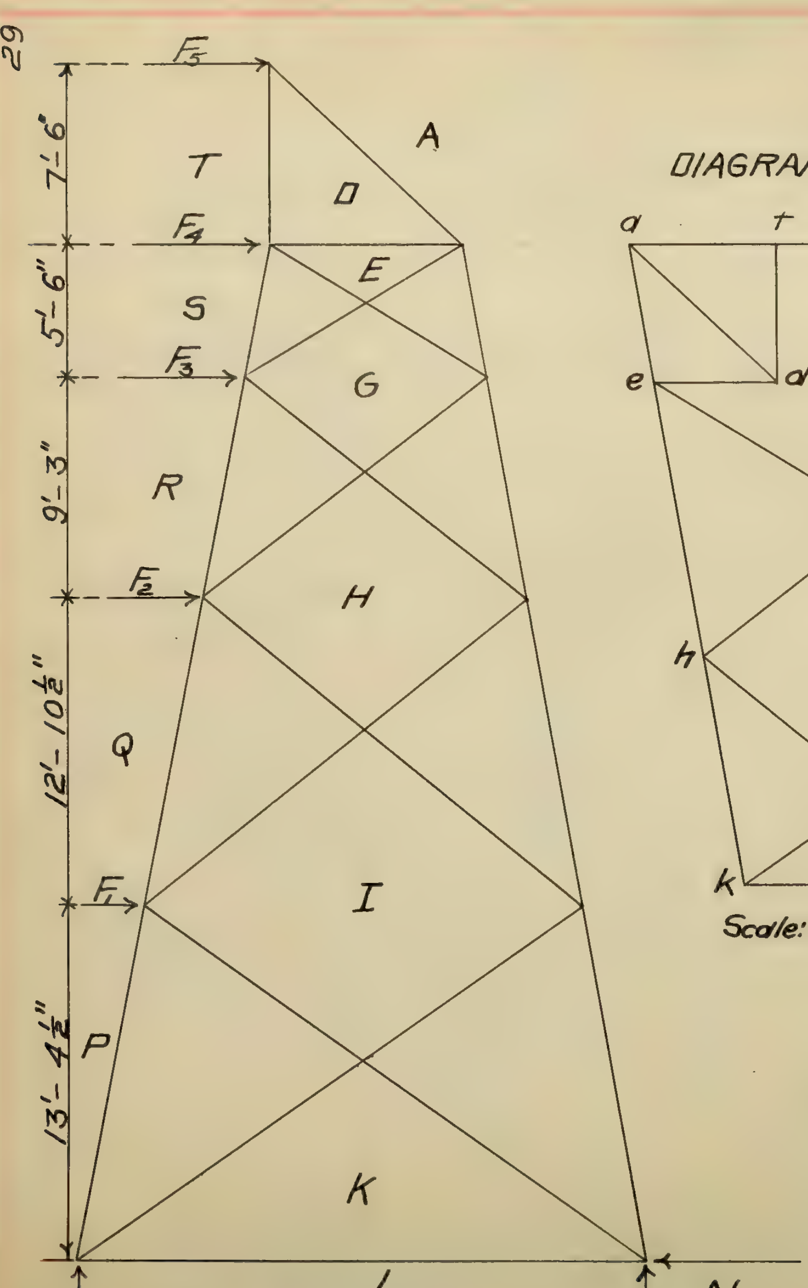


STRESSES BY LEAST WORK

Mem.	l	a	S'	u	$\frac{ul}{a}$	$\frac{S'ul}{a}$	$\frac{u^2l}{a}$	$S_r u$	S
ca	68	37.72	+236,000	−.50	−.90	−212,000	+.45	−17,100	+218,000
bc-m	96	11.76	+43000	−.95	−7.75	−333,000	+.70	−32,600	+10,400
be	128	11.76	−45,000	+1.00	+10.8	−485,000	+10.80	+34,300	−10,700
ec-m	68	37.72	+24000	−.50	−.90	−21600	+.45	−17,100	+6900
ed	128			+1.00					
cg-m	112	37.72	+24000	+.50	+1.49	+35,800	+.74	+17,100	+41,100
gd	180	11.76	000	−.85	−13.0	000	+11.00	−29200	−29200
dh	180	11.76	+38,000	−.85	−13.0	−494,000	+11.00	−29,200	+8,800
hc	112	37.72	+190,000	+.50	+1.49	+283,000	+.74	+17,100	+207,100
ck-m	156	37.72	+65000	−.40	−1.65	−107,000	+.66	−13700	+51,300
kj	246	11.76	−28,000	+.64	+13.4	−375,000	+.66	+22000	−6,000
jl	246	11.76	000	+.64	+13.4	000	+.66	+22,000	+22000
lc	156	37.72	+190,000	−.40	−1.65	−314,000	+.66	−13,700	+176,300
com	166	37.72	+65000	+.30	+1.32	+86,000	+.39	+10,300	+75,300
on	307	11.76	000	−.45	−11.7	000	+5.27	−15,400	−15400
np	307	11.76	+20,000	−.45	−11.7	−234,000	+5.27	−15,400	+4,600
pc	166	37.72	+160,000	+.30	+1.32	+211,000	+.39	+10,300	+170,300
qr	285	11.76	−30,000	+.35	+7.30	−219,000	+2.56	+12,000	−18,000
						−2178000	+52.50		

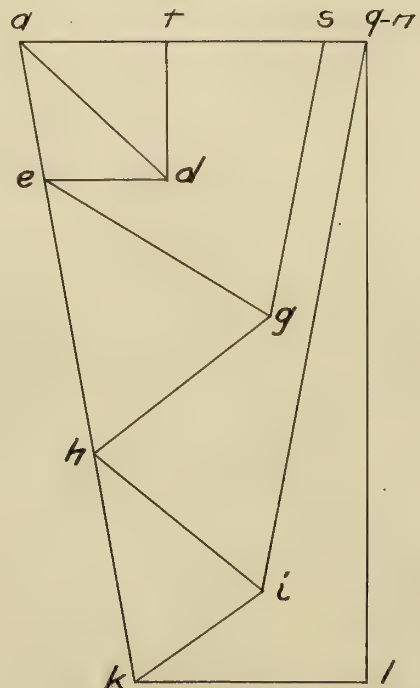
$$\frac{l_r}{a_r} = \frac{128}{11.76} = 10.9 \qquad S_r = -\frac{-2178000}{52.50+10.9} = +34300$$

Fig. 9. LIVE LOAD STRESSES, ROCKER BENT T

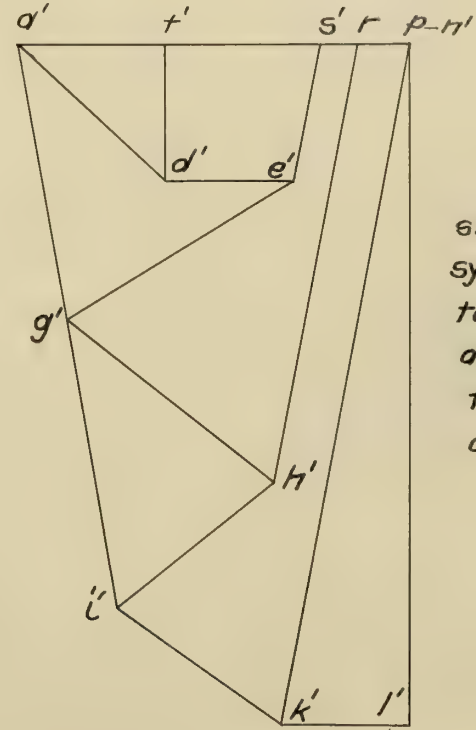


Scale: $\frac{1}{8}$ inch = 1-Ft.

DIAGRAMS FOR MAXIMUM WIND STRESSES



Scale: 1-inch=10,000 lb



Scale: 1-inch = 10,000 lb.

NOTE,
It was assumed that each system of bracing took stress F_5 & F_4 and their respective forces down the side of the bent.

WIND FORCES, TRAIN OFF

F_5	$\frac{60}{4} \times 500$	7,500
F_4	$\frac{60}{4} \times 500 + \frac{5.5}{2} \times 200$	8,050
F_3	$\frac{5.5+9.25}{2} \times 200$	1,475
F_2	$\frac{9.25+12.87}{2} \times 200$	2,212
F_1	$\frac{12.87+13.37}{2} \times 200$	2,624

Fig. 10.

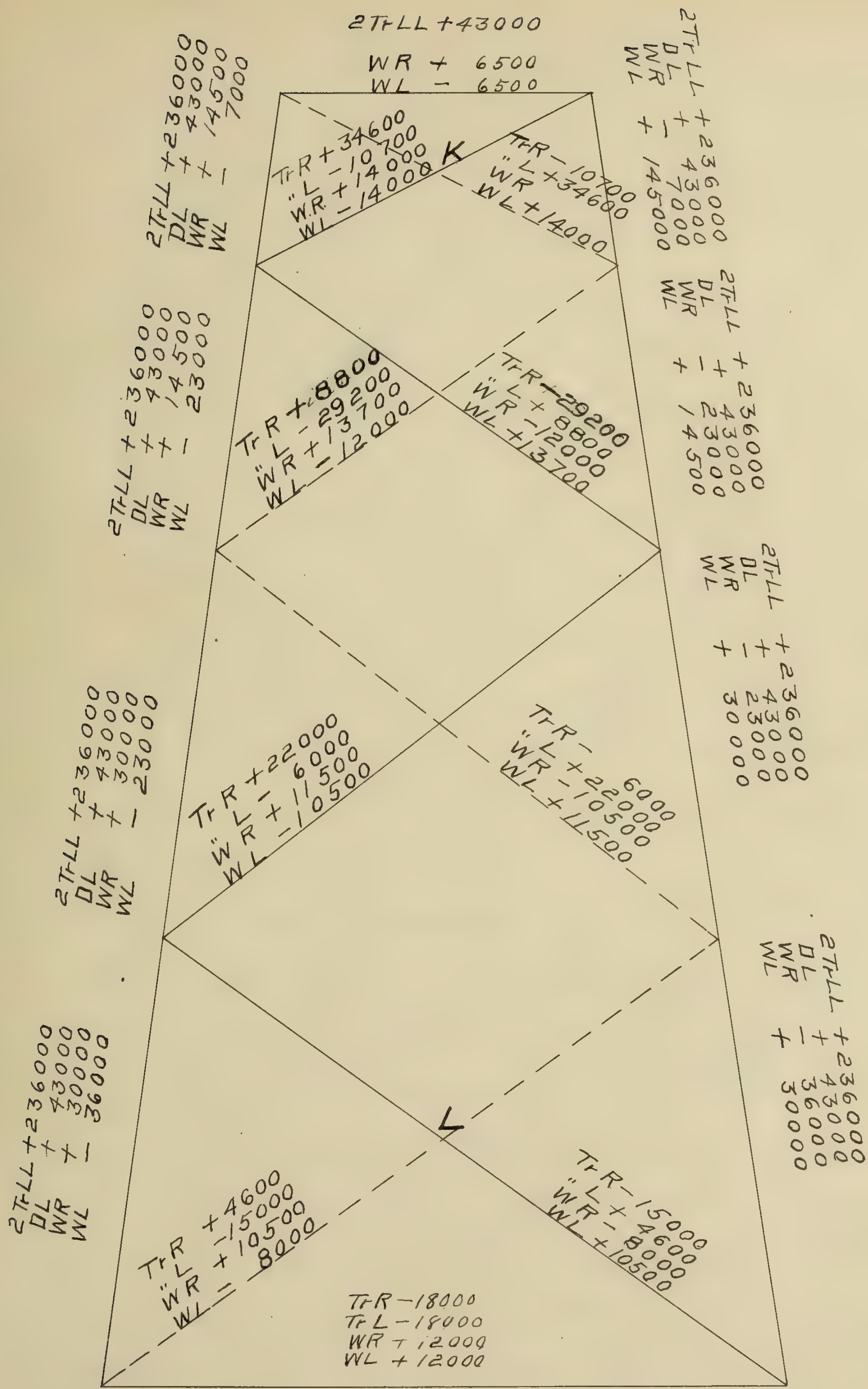


Fig. 11. STRESS SHEET FOR ROCKER BENT T

the least radius of gyration of one of the channels is 3.66. The maximum direct stresses in the member are 73,600 lb. compression and 32,400 lb tension. These values include stresses due to impact. The unit stress, p, for tension = 15000 lb per sq. in., and for compression, $r = 3.66$, $l/r = 35$ and therefore $p = 13,750$ lb. per sq. in. The area required for tension is $32,400/15,000 = 2.16$ sq. in., and for compression $73,600/13,750 = 5.35$ sq. in. The rivet spacing in the end connections is the same as that shown in Fig. 6, p 15, consequently 2.6 sq. in. must be deducted from the section gross area making 10.11 sq. in. required. The given area is 11.76 sq. in. which gives 16.3% excess area for the member. The number of rivets required is 11, while 28 are used.

THE DIAGONAL L

This member is located in the bottom panel of the bracing of the bent, Fig 10, having the same section as the one investigated above and a length of 306 inches. The maximum direct stresses in the member are 89,500 lb compression and 34,000 lb tension. These include stresses due to impact. The unit stress, p, for tension = 15,000 lb. per sq. in. and for compression $r = 3.66$, $l/r = 83.5$ and therefore $p = 9,900$ lb. per sq. in.

The area required for tension is 34,000/15,000 = 2.26 sq. in. and for compression 89,500/9,900 = 9.05 sq. in. The rivet spacing in the end connections is the same as that shown in Fig. 6, consequently 2.6 sq. in. must be deducted, making a total required area of 13.91 sq. in. The given area is 11.76 sq. in., which gives 15.4 % deficiency in area for this member. The number of rivets required is 14, while 28 are used. I would not give much weight to the deficiency in the cross-section of this member, in as much as the chances of the maximum wheel load coming on both tracks at the same time and during a heavy wind are very remote. An investigation of the diagonals in the second and third panels of the bracing show them to have sufficient section.

CONCLUSION

In conclusion I would say that all the members of this structure have sufficient cross-section to safely resist the maximum stresses that can be caused in them by reason of the specified loads being brought ~~upon~~ the viaduct. The design is so simple and economical, that I think 3.8¢ per pound would be a fair average price for the structure complete with

two coats of paint included. The estimated weight is as follows.

Girders (complete)	1980 000 lb.
Towers "	632 700 "
Total	2,612,700

The estimated cost is 2,612,700 lb. × 3.8¢ per pound equals \$100,000